METROPOLITAN ENGINEERS

410 WEST HARRISON • SEATTLE, WASHINGTON 98119 • AT 4-5110

A JOINT VENTURE OF
BROWN AND CALDWELL
CAREY AND KRAMER
HILL & INGMAN
R. W. BECK AND ASSOCIATES

June 2, 1965

W210E-19

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FINAL REPORT
FOUNDATION INVESTIGATION
INTERBAY PUMPING STATION (W210E)
SEATTLE, WASHINGTON

INTRODUCTION

This report summarizes the results of the foundation investigation conducted for the proposed Interbay pumping station. As shown by the Vicinity Map, Plate 1, the proposed pumping station will be located near the intersection of West Garfield Street and Alaskan Way West. The proposed structure will be rectangular in shape below ground level with outside dimensions approximately 63 feet by 79 feet. The base slab will be at about elevation 77±*. A silencer room with base floor at elevation 105 will project some 7 feet beyond the northwest station wall. A 102-inch influent sewer will enter the southwest wall at invert elevation 89± and two 36-inch effluent force mains will discharge through the northwest wall at about elevation 101. The existing ground surface is at about elevation 111 and will be raised by filling to elevation 113.

The purpose of this investigation was to explore the existing soil conditions at the proposed location and to obtain sufficient information to permit the design of the facility. The results as presented herein were discussed with our design engineers during the course of the investigation.

*Elevation 100 = Mean Sea Level

FIELD EXPLORATIONS AND LABORATORY TESTING

The field explorations were carried out by the drilling of nine test borings at the locations shown on the Vicinity Map. Borings 1 through 7 were drilled with cable-tool type drilling equipment to depths varying from 83 to 98 feet below existing ground surface. Representative undisturbed samples of the soils encountered in these borings were secured for visual examination and laboratory testing. These samples were taken using a 3.25-inch O.D. split-barrel sampler driven by means of a 500-pound drop weight falling approximately 20 inches. Borings 8 and 9 were drilled with 7-inch O.D. hollow-core auger drilling equipment. Disturbed samples were taken using the standard penetration test method, i.e., 2-inch O.D. Gow split-spoon sampler driven with a 140-pound drop weight falling 30 inches. The soils encountered in the borings are shown graphically on the boring logs on Plates 3 through 11. The blow counts required to drive the samplers one foot are shown at the respective sample elevations. (Double the blows for the 2nd 6 inches of driving.)

Ground water conditions were observed during the drilling, after sampling and overnight. Further, four piezometers, 6-inch long porous stone with 1/2-inch I.D. plastic riser, were installed in the upper and/or lower sand and gravel layers of B2, B3 and B7. This information is also included on the boring logs.

Representative undisturbed samples were subjected to various laboratory tests. The in-place density and moisture content of undisturbed samples obtained from the borings were determined. Atterberg limit tests were performed on selected clay and silt samples for identification and correlative purposes. Strain-controlled unconfined compression tests were also performed on certain clay and silt samples to determine the physical strength of the cohesive soils. These test results are shown at the left of the respective boring logs. In addition, consolidation tests were performed on selected clayey samples as an aid in predicting settlement behavior. These test results are presented on Plates 14 through 18. Sieve analyses were also run on selected granular samples from major water-bearing zones as an aid in evaluating aquifer characteristics and possible dewatering methods. The resulting grain-size curves are shown on Plates 19 through 26. It should be pointed out that the sieve tests were performed on samples taken with a 2½-inch I.D. sampler. Thus material larger than 2½ inches may be present in the natural deposit although it is not indicated on the gradation curve.

SUBSURFACE CONDITIONS

The field explorations disclosed that, at the boring locations, sand and gravel fill, containing silt, shells and debris, blankets the site down to about elevation 95. Immediately beneath the fill, beach deposits of predominately fine sand with gravelly layers and shell fragments extend down to about elevation 82 on the west and elevation 86 on the east. Soft marine deposits consisting of silts with some fine sands and abundant shells were encountered below the surface sand and gravel down to elevation 30 at B7 and elevation 66 at B2. A water-bearing sand and gravel layer underlies the soft silt to elevations ranging from elevation 19 to 52, except at B8 where the boring terminated in sand and gravel at elevation 15. In the remaining borings, firm glacially preconsolidated clay was encountered to the depths explored. Near the structure location, the bottom of the soft silt ranged from elevation 32 to elevation 46 with the base of the lower sand and gravel varying from elevation 22 to elevation 38, except at B8. In general, the sampler blow counts and laboratory tests indicate that the silt is softer and thus somewhat weaker and more compressible in the middle of the layer between about elevation 70 and elevation 60.

During the drilling the highest ground water level in the surface sand and gravel layer was measured at about elevation 102. The highest artesian pressure head observed in the lower water-bearing sand and gravel layer during the drilling was at elevation 101. Subsequent to the drilling, the water levels in the piezometers were measured at 2-hour intervals on November 17, 1964, and May 13, 1965. The results, together with the tidal curves for the Seattle area, are plotted on Plates 12 and 13. These observations recorded ground water levels as high as elevation 104 in the surface sand and gravel and elevation 106.4 in the lower artesian aquifer. Both water-bearing zones showed tidal effects with the lower zone having a more pronounced response.

DISCUSSION AND RECOMMENDATIONS

On the basis of the conditions disclosed by this investigation, it is concluded that construction of the Interbay pumping station at the proposed site is feasible provided piling is used to support the structure. The soft compressible silts, extending some 30 feet to 45 feet below the base slab, are inadequate for supporting the structure without excessive settlement should the structure be supported directly on the soil. Therefore it is recommended that the pumping station be supported on piling driven into the lower sand and gravel or the underlying firm clay.

Because of the difference in excavated and backfill soil weights, as well as additional site fill, some consolidation of the underlying soft silt will occur. This will cause downward movement of the backfill relative to the station walls. It is estimated that this down-drag could approach 13 kips per lineal foot of wall depending on the size of the excavation and future ground water levels. Therefore, the piling will have to develop capacity to support this down-drag load in addition to the structural loads. It is understood that the combined loadings result in loads on the order of 80 tons per pile for the completed structure. Displacement piles such as prestressed concrete or pipe piles will develop capacities of this magnitude at shorter penetrations than nondisplacement piles such as H-piles. The upper surface of the recommended supporting soils range from elevation 32 to elevation 46 in the borings at and near the structure location. The thickness of the sand and gravel layer varied from 8 to 17 feet. While the penetration of the pile tips will be variable across the site, it is believed that, in general, the required bearing will be obtained with relatively shallow penetration. Thus the piles will be essentially end bearing rather than friction piles. Further, the piles will be penetrating into an artesian aquifer. Depending on the hydrostatic head in the aquifer, upward seepage could and probably will occur along the piles. Therefore the piling should not be relied upon to resist uplift either during or following construction. If uplift is a consideration for the completed structure, resistance could be obtained by projecting the base slab beyond the station walls and utilizing the weight of the overlying backfill. The backfill weight may be taken as 130 pounds per cubic foot above, and 70 pounds per cubic foot below, the appropriate design water table.

It is not possible to accurately predetermine driving characteristics for these conditions. To allow for variation in the density and thickness of the sand and gravel layer, as well as possible lower driving resistance in the underlying clay should the piles penetrate through the sand and gravel, it is recommended that the pile tip elevations be assumed at elevation 15 for lump sum bidding purposes. In view of the size and weight of piling required, suitable driving equipment should be used to assure developing adequate bearing capacity. If prestressed concrete piling are used, it is recommended that the pile hammer be capable of delivering at least 24,500 foot-pounds of energy per blow; for pipe piles, the pile hammer should be capable of delivering at least 18,000 foot-pounds of energy per blow. Questionable or insufficient pile penetration of the interior piles may result if the perimeter piles are driven first. Therefore, it is further recommended that either the piles be installed from the center outwards or that the pile driving proceed from one end of the foundation towards the opposite end.

Construction of the pumping station will require excavation below the invert elevation of the 102-inch influent sewer. Since it is unlikely that this can be accomplished without disturbing the supporting soils, it is recommended that at least 15 to 20 feet of this sewer be constructed at this time and that the sewer be supported on piling in the same manner as the pumping station. Since the pipe will be rigidly supported, the backfill loading will be greater than that due to the soil directly over the pipe. This has been considered in the pipe load analysis and it is recommended that the 102-inch influent sewer be designed for a backfill load of 26,000 pounds per lineal foot. Twin 36-inch I.D. force mains will also be constructed as part of this contract. The borings disclosed that at least 15 feet of medium dense sandy soils underlie the force main alignments. It is understood that nominal settlements can be tolerated. If this is so, piling are not needed for support of the force mains provided at least 12 inches of compacted bedding is provided beneath the pipe. Estimates have been made of the backfill loading on the pipe and it is recommended that the force mains be designed for a backfill load of 6,000 pounds per lineal foot of pipe. It is understood that the force mains will be designed as flexible steel pipe. Provided a granular fill compacted to 95 per cent of the ASTM maximum dry density is placed for 12 inches below to 6 inches above and extended for one diameter each side of the pipe, a modulus of passive resistance equal to 700 psi may be used in the pipe design. The backfill around and above the pipe should be compacted in contact with the adjacent natural soil. It is also recommended that flexible couplings be provided at the pipe joints, particularly at the pumping station excavation limit.

The pumping station walls are relatively rigid and are restrained at the top and bottom as well as at intermediate points. Sufficient yielding of the walls are not expected to take place which would reduce the lateral soil pressure to its active value. Further, the intensity distribution is essentially trapezoidal rather than triangular. These factors were considered in evaluating the lateral pressures which will be imposed against the station walls. In view of the tidal effect on ground water levels as observed in the piezometers, a ground water level at elevation 109.5 was used which takes into account the highest recorded tide and the difference in density of fresh and sea water. The results of this analysis are presented on Plate 2, Wall Pressure.

Except for sidewalks and paving, the backfill around the pumping station will not be used for load support. However, because of the depth of backfill required, settlement within the backfill itself could range from 6 to 12 inches if only moderately compacted granular material is used. If this magnitude of settlement cannot be tolerated, granular material compacted to 95 per cent of the ASTM maximum dry density should be specified. As mentioned above, some consolidation of the underlying silt will occur due to backfill weights as well as additional site fill. It is estimated that this will range from fractions of an inch adjacent to the station to about one inch at the excavation periphery - provided the excavation limits do not extend about 10 feet beyond the station wall.

Some site fill will be required to raise the site to final grade. It is recommended that Type B granular fill compacted to 95 per cent of the ASTM maximum dry density be used. In order to provide a satisfactory subbase beneath paving, it is recommended that the upper 12 inches beneath those areas be compacted to 100 per cent of the maximum dry density. If this is done, satisfactory paving behavior may be anticipated for any foreseeable wheel loading.

Cognizance must be taken of the soil and ground water conditions in the construction of the pumping station as well as in the design. The effect of the method of construction on adjacent existing facilities should also be considered. The closest structure to the pumping station is the Garfield Street bridge. According to Seattle Engineering Department records, the original concrete structure was constructed in 1929 with the elevated lanes supported on piles driven to about elevation 49. The west abutment wall of the sloping ramp to 15th Avenue West is also pile-supported. The north and south walls are concrete gravity retaining walls with a single pile at the toe on 7- to 5-foot centers for part of their length. The pile penetrations are not known. The ramp roadslab is reported to be supported on backfill between the walls. Numerous cracks are visible in the original construction. Additional lanes were constructed on the north in 1957 and are supported on piling reportedly driven to firm bearing.

Construction of the pumping station will require excavating some 40 feet below existing ground surface. Approximately the lower one third of the excavation will be in the soft marine deposits. Unsupported open cut is not believed practicable because of the large work area needed to provide safe excavation slopes. Thus, it is assumed that a contractor would employ a sheeted excavation with the sheeting extending a sufficient distance below the excavation bottom to prevent the movement of adjacent soft soils into the excavation. Calculations indicate that the pressure head in the underlying artesian aquifer will endanger the excavation bottom unless this hydrostatic head is counteracted or reduced. A preliminary study was made of the effect on the adjacent bridge of reducing the pressure head to elevation 67 - or slightly below the level of the excavation. Because of its proximity, a similar drawdown was assumed to extend beneath the bridge ramp. Depending on the assumptions made as to time rates (i.e., portion occurring as primary consolidation), it was calculated that the bridge ramp would settle 6 to 12 inches in 12 to 16 months if dewatering continued for that period.

It appears that several construction methods, or combinations of methods, are available which would eliminate or substantially reduce the possibility of damaging settlements. Among these are: simultaneously recharging the water-bearing zones between the adjacent structures and the sheeted excavation while constructing the station; excavating and driving piling in the wet and pouring a tremie seal of sufficient thickness to counteract the uplift; partial pressure head reduction combined with subaqueous excavation, piledriving and a partial tremie seal using an underdrain system to reduce uplift on the tremie due to seepage up along the foundation piles; sealing off the aquifer by extending the cofferdam sheet piling into the underlying firm clay, partial pressure relief within the cofferdam and either subaqueous or dry construction.

It is also possible that a contractor may elect to use a sonic pile hammer to eliminate vibration caused by driving sheet piling and/or foundation piling. At the present time, we do not have sufficient information to establish penetration criteria without substantiating pile load tests. However, should this be proposed, it is believed that adequate criteria can be obtained by driving one or two piles to required bearing using a steam hammer and load testing two piles installed to similar elevations with a sonic hammer.

The selection of the construction method and details would depend on the contractor's experience and equipment. Since more than one method appears available which would accomplish the construction without damage to existing facilities, the choice and responsibility may be left to the contractor.

METROPOLITAN ENGINEERS

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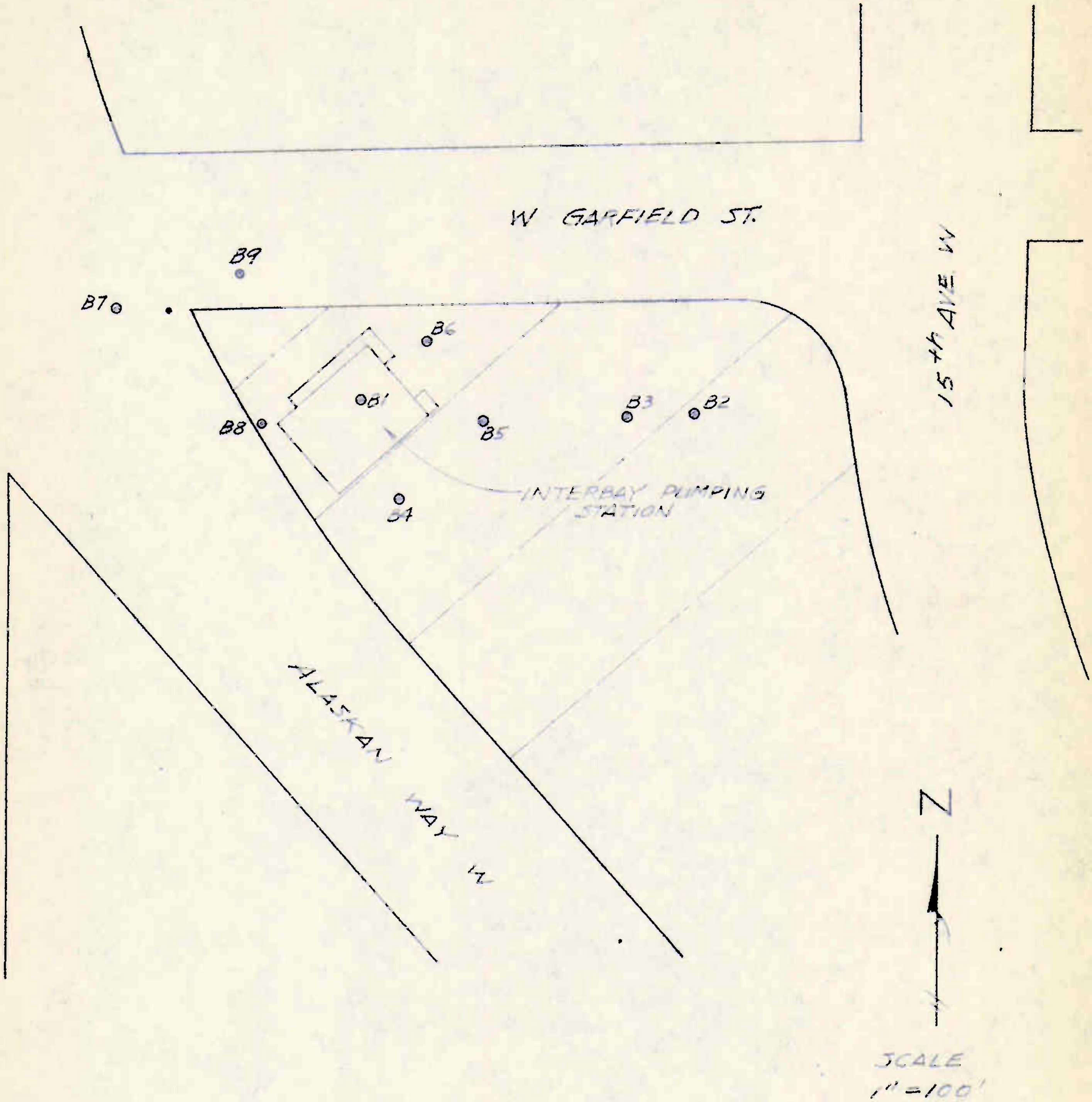
ERM/CHW:ab
Attachments - 26 plates

CALCULATION SHEET
METROPOLITAN ENGINEERS
SEATTLE, WASHINGTON



DATE	BY	JOB NO.	TITLE	PLATE
	CHW	W210E	VICINITY MAP	1

CALCULATION SHEET
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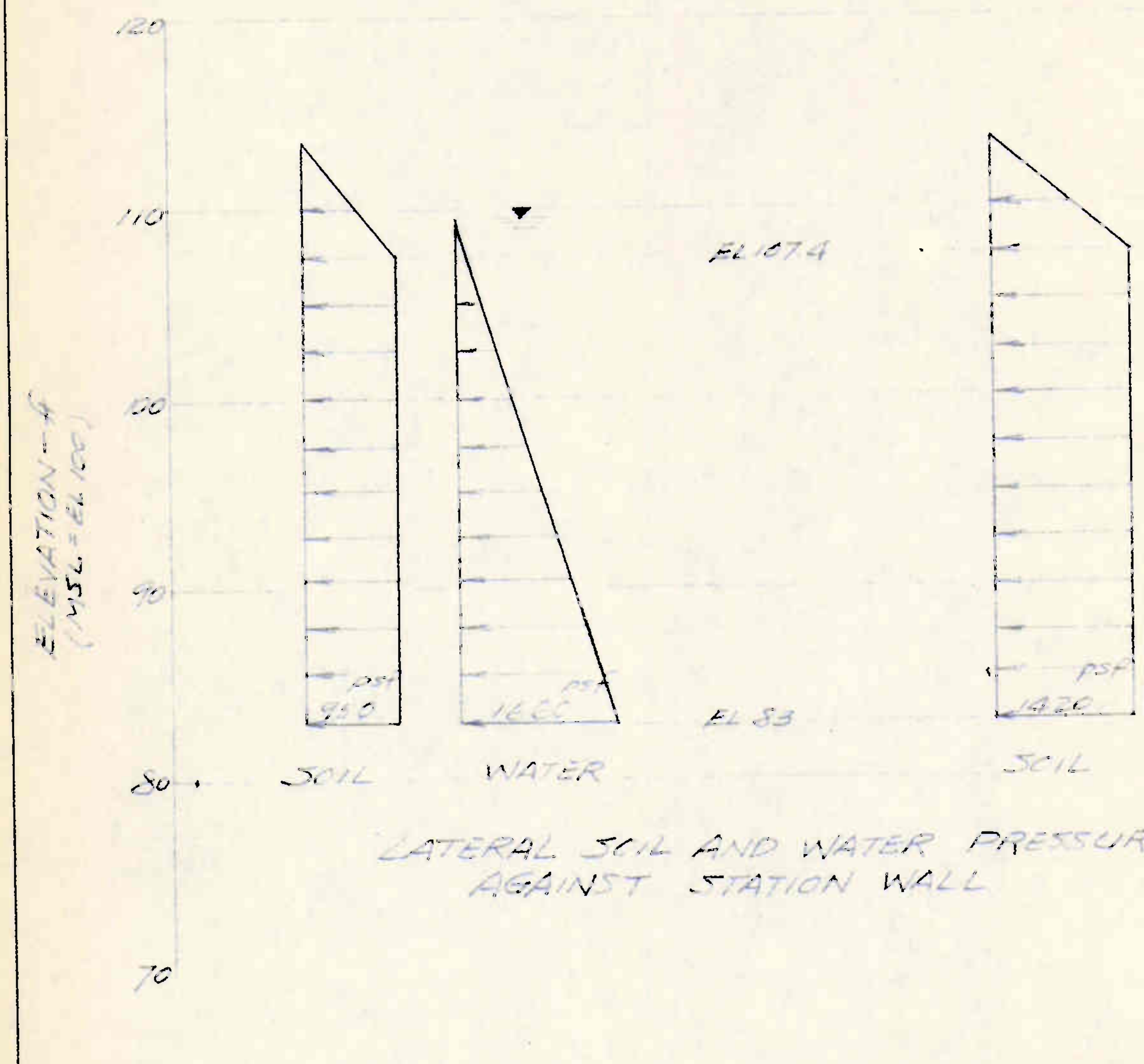
(A) WATER LEVEL @ EL 109.5 (B) DEWATERED



LATERAL SOIL AND WATER PRESSURES
AGAINST STATION WALL

DATE	BY	JOB NO	TITLE	PLATE
2-11-65	CHW	W210E	WALL PRESSURES	2

CALCULATION SHEET
METROPOLITAN ENGINEERS
SEATTLE, WASHINGTON

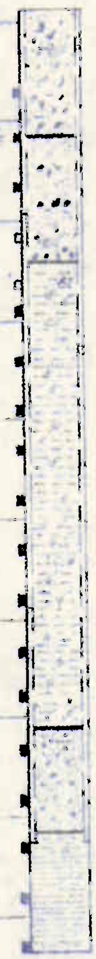
MSL = EL 100

BORING B1

ELEVATION 111.4
DATE DRILLED 4-13 to 16-1964

ELEVATION - ft (MSL = EL 100)

w %	γ_d pcf	LL %	PL %	q_u ksf	N
[illegible]	47				40
24.6	114				16
13.6	125				
					36
					2
30.8	86	39.3	25.5		2
41.6	83	41.2	25.7		1
41.0	77	53.3	29.5		3
53.1	66	84.3	35.3		2
43.5	75	49.1	28.4		4
33.8		38.2	25.7		8
25.1	100	34.1	21.9		4
43.2	76	55.5	31.6		4
37.6	85	33.2	24.0		2
13.9					12
9.8	131				75
34.5	89	59.1	32.0		23
27.4	91	46.9	27.5		36
34.1	91	60.7	32.3		48



BROWN GRAY SAND AND GRAVEL WITH SOME SILT (FILL?) CONTAINS DEBRIS

CASING AND BORING @ EL 100, OVERNIGHT LITTLE WATER IN BORING

GRAY FINE SAND WITH SAND AND GRAVEL LAYERS AND SHELL FRAGMENTS

AFTER TAKING SAMPLE WATER LEVEL ROSE FROM EL 97 TO EL 98 IN 1/2 HOUR CONTAINS SCRAP METALS

GREEN GRAY SILT WITH SHELL FRAGMENTS - SOFT

CONTAINS OCCASIONAL DECAYED WOOD

GRADES CLAYEY

CONTAINS SOME FINE SAND

BORING @ EL 56, CASING @ EL 72, OVERNIGHT W.L. @ EL 73

GRADES TO MODERATELY FIRM

SOME THIN FINE SAND LAMINATIONS OR LENSES

GRAY SILTY SAND AND GRAVEL WITH OCCASIONAL SHELL FRAGMENTS

GRADES CLEAN (GRAVEL TO 3")

SEE NOTE 1 FOR WATER LEVEL READINGS

GRAY CLAY WITH SOME SILT -- VERY FIRM

NOTES:

1. CASING @ EL 39, BORING @ EL 36, AFTER TAKING SAMPLE WATER LEVEL ROSE FROM EL 47 TO EL 49 IN 1/2 HOUR, CASING AND BORING @ EL 32, WATER LEVEL WAS @ EL 57 AND RISING. CASING @ EL 32, BORING @ EL 31 AFTER TAKING SAMPLE WATER LEVEL WAS @ EL 96 AND RISING TO EL 97 IN 1/2 HOUR.
2. AFTER COMPLETION CASING REMOVED AND BORING BACKFILLED WITH NATIVE MATERIAL

LEGEND

- MEASURED HIGHEST WATER LEVEL
- ■ SAMPLE TAKEN WITH 2 1/2" ID SPLIT-BARREL DRIVE SAMPLER, □ SAMPLE MISSED
- w MOISTURE CONTENT IN PER CENT OF DRY WEIGHT γ_d = DRY DENSITY
- N NUMBER OF BLOWS PER FOOT OF PENETRATION, DRIVING WEIGHT = 300 lbs FALLING DISTANCE = 20" 1 SAMPLER OD = 3 1/4" LL = LIQUID LIMIT PL = PLASTIC LIMIT q_u = UNCONFINED COMPRESSIVE STRENGTH
- DEPTH TO WHICH CASING WAS REQUIRED DURING DRILLING

DATE	BY	JOB NO	TITLE	PLATE
	CHW	WENCE	LOG OF BORING	3

CALCULATION SHEET

METROPOLITAN ENGINEERS

SEATTLE, WASHINGTON

BORING B2

ELEVATION 109.8

DATE DRILLED 4-16, 17 & 20-64

ELEVATION - ft (MSL = EL 100)

120, 110, 100, 90, 80, 70, 60, 50, 40, 30, 20

W %	γ_d pcf	LL %	PL %	N
17.9	108			21
7.4	133			24
25.5	104			12
11.7	123			28
39.0	84	39.2	26.2	4
37.2	80	34.2	25.6	2
52.2	68	[illegible]	34.6	2
				10
				4
14.8	125			10
				1
21.4	110			16
32.6	91	59.1	29.2	24
36.3	89	63.4	32.6	20
32.6	91	60.0	29.0	22
35.0	89	75.1	32.5	22
23.7	75	48.3	26.7	28

BROWN GRAY FINE-MED SAND WITH SOME GRAVEL AND SHELL FRAGMENTS

GRADES GRAVELLY

GRAY FINE SAND WITH SAND AND GRAVEL LAYERS AND SHELL FRAGMENTS

CASING AND BORING @ EL 88, OVERNIGHT WATER LEVEL @ EL 102

GREEN GRAY SILT WITH SHELL FRAGMENTS - SOFT

GRADES CLAYEY

GRAY SILTY SAND AND GRAVEL

CONTAINS OCCASIONAL DECAYED WOOD

WATER ENTERING (GRAVEL TO 3") SEE NOTE 1

LAMINATED BROWN F-VF SAND AND SILT LAYER (60° DIP) - DENSE

GRAY CLAY WITH SOME SILT - VERY FIRM

CONTAINS OCCASIONAL PEBBLES

SLICKENSIDES

CONTAINS VERY THIN SILT LAMINATIONS OR PARTINGS

NOTES:

1. AFTER BORING ADVANCED TO EL 42 WITH CASING @ EL 50. CASING WAS PULLED UP TO EL 55 AND BORING FILLED WITH WATER TO EL 55. 3 DAYS LATER WATER LEVEL @ EL 101

2. AFTER COMPLETION, CASING WAS PULLED UP TO EL 55, HOLE BACKFILLED WITH SAND AND GRAVEL TO EL 55. A PIEZOMETER (6" POROUS STONE WITH ½" ID RISER) WAS INSTALLED WITH THE TIP @ EL 55. MEDIUM SAND WAS PLACED BETWEEN EL 55 AND EL 60. THE REMAINDER OF THE HOLE WAS BACKFILLED WITH NATIVE MATERIAL EXCEPT ONE SACK OF SLURRY CEMENT PLACED @ EL 70

DATE	BY	JOB NO.	TITLE	PLATE
	CHW	W210E	LOG OF BORING	4

CALCULATION SHEET

METROPOLITAN ENGINEERS
SEATTLE, WASHINGTON

BORING B3

ELEVATION 109.5
DATE DRILLED 4-21 & 22-64

ELEVATION - ft (MLLW ... 100)

W	γd	L.L.	P.L.	N
13.6	118			34
				16
19.2	108			18
8.1	147			13
				2
				5
32.3	85	40.8	26.1	2
57.5	71	[illegible]	32.6	4
48.2	73	56.5	31.3	2
				10
26.2	113			43
				48
9.0	128			28
16.0	102			22
23.1	96	47.3	29.5	26
32.5	92	59.4	28.[illegible]	20
				48
37.4	85	57.6	33.0	1[illegible]

BROWN GRAY FINE-MED SAND WITH GRAVEL AND SHELL FRAGMENTS

GRAY FINE SAND WITH SAND AND GRAVEL LAYERS AND SHELL FRAGMENTS

GREEN GRAY SILT WITH SHELL FRAGMENTS - SOFT

GRADES CLAYEY

GRAY SAND AND GRAVEL WITH SOME SILT

CONTAINS OCCASIONAL DECAYED WOOD

CASING @ EL 55, BORING @ EL 58, OVERNIGHT WATER LEVEL @ EL 100

CONTAINS SOME SHELL FRAGMENTS (GRAVEL TO 3")

LESS GRAVEL

CONTAINS SOME BROWN PEAT

GRAY CLAY WITH SOME SILT - FIRM

NOTES

1. CASING REMOVED AND BORING BACKFILLED WITH PEA GRAVEL EXCEPT A BAG OF CEMENT WAS PLACED @ EL 77
2. A PIEZOMETER (6" POROUS STONE WITH ½" ID RISER) WAS INSTALLED WITH THE TIP @ EL 92

DATE	BY	JOB NO.	TITLE	PLATE
	CHW	N-2[illegible]E	LOG OF BORING	5

CALCULATION SHEET
METROPOLITAN ENGINEERS
SEATTLE, WASHINGTON

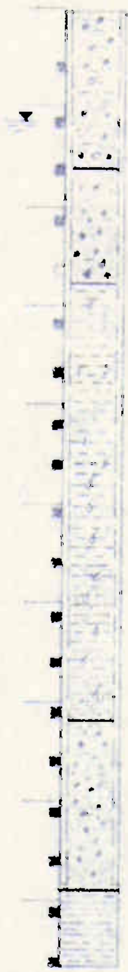
BORING B4

ELEVATION 109.5
DATE DRILLED 9-24,25,28-64

ELEVATION - ft (MSL = EL 100)

Elevation	W %	γ_d pcf	q_u ksf	N
	5.9	119		26
	15.4	114		24
	24.4	107		10
	20.2	108		40
				1
	30.9	89	0.61	4
	33.2	88	0.64	1
	45.7	72		2
	35.0	81	0.72	2
	46.0	81	0.68	4
	22.1	105	0.99	4
	49.2	72	1.35	1
	38.9	80	1.21	4
	21.3	106	0.86	8
	10.1	122		85
	6.9	128		80
	11.2	122		80
	31.9	90	5.40	30
	32.4	89	5.30	30

Elevation scale: 120, 110, 100, 90, 80, 70, 60, 50, 40, 30, 20, 10



BROWN GRAY SAND AND GRAVEL (FILL)

CONTAINS SOME SILT AND SHELL FRAGMENTS

GRAY SILTY FINE SAND WITH SOME SHELL FRAGMENTS AND OCCASIONAL GRAVEL
GRADES GRAVELLY

GREEN GRAY SILT WITH SHELL FRAGMENTS

CONTAINS FINE-MED SAND

CONTAINS SOME SAND

GRAY SAND AND GRAVEL WITH SOME SILT AND SHELL FRAGMENTS - DENSE

GRADES LESS GRAVEL AND NO MORE SHELLS

CASING AND BORING @ EL 25. 3 DAYS LATER WATER LEVEL @ EL 98

GRAY CLAY WITH SOME SILT - VERY FIRM

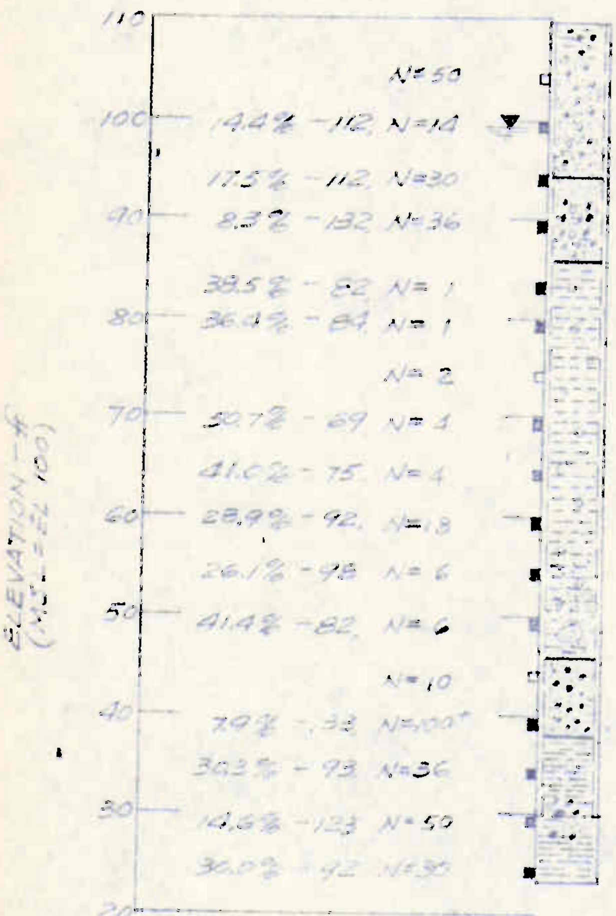
NOTE: CASING REMOVED AND BORING BACKFILLED WITH PEA GRAVEL

DATE	BY	JOB NO.	TITLE	PLATE
	CAW	W 2102	LOG OF BORING	6

CALCULATION SHEET
METROPOLITAN ENGINEERS
SEATTLE, WASHINGTON

BORING B5

ELEVATION 109.9
DATE DRILLED 9-28,29,30-64



BROWN GRAY SAND AND GRAVEL (FILL)

CONTAINS SHELL FRAGMENTS
CONTAINS SILT
GRAY LAYERED FINE SAND WITH SOME SILT AND SILTY SAND AND GRAVEL
CONTAINS SHELL FRAGMENTS
GREEN GRAY SILT WITH SHELL FRAGMENTS - SOFT

CONTAINS SOME SAND

WOOD
GRAY SAND AND GRAVEL
CASING @ EL 38, BORING @ EL 40, OVERNIGHT WL @ EL 100
GRAY SILTY CLAY - VERY FIRM

CONTAINS SOME SAND AND GRAVEL

NOTES: 1) CASING REMOVED AND BORING BACKFILLED WITH PEA GRAVEL

2) 14.4% - 112 INDICATES MOISTURE CONTENT = 14.4%, DRY DENSITY = 122 PCF

DATE	BY	JOB NO.	TITLE	PLATE
	CHW	W2KE	LOG OF BORING	7